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Reinforced Concrete Framed Building Strengthened by Prestressing

Cadres en béton armé renforcés par la précontrainte

Rahmentragwerke aus Stahlbeton verstärkt durch Vorspannung

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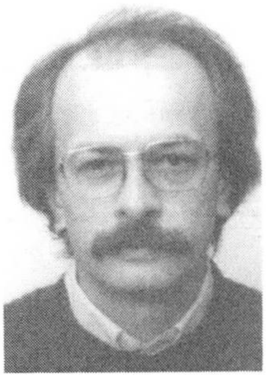

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SUMMARY

Due to inadequate structural design and reinforcement detailing, a three-story reinforced concrete framed structure has suffered excessive cracks and deflection, followed by local failure of columns. The structure's bearing capacity has been improved by shoring each frame beam with prestressing tendons conducted below the beam soffit, in a "two-chord" system. Analysis of causes, details of strengthening and results of deflection measurements are presented in the paper.

RÉSUMÉ

Les cadres en béton armé d'un bâtiment à trois étages ont été gravement fissurés et déformés par suite de calcul et de disposition de l'armature constructive inadéquats. Le renforcement de la structure portante est effectué par des câbles de précontrainte extérieurs, en combinaison avec des plaques métalliques collées. L'analyse des causes, les détails de renforcement et les résultats de mesure des flèches après la mise de précontrainte sont présentés.

ZUSAMMENFASSUNG

Die unangemessene Berechnung und Bewehrung des Rahmentragwerks aus Stahlbeton eines dreistöckigen Gebäudes hat übermässige Risse und Durchbiegungen hervorgerufen. Die Verstärkung des Tragwerks wurde durch Vorspannung der Rahmen mit Aussenspanngliedern durchgeführt, so dass zweigurtige Konstruktionssysteme gebildet sind. Die Analyse von Ursachen, die Verstärkungseinzelheiten und die Ergebnisse der Durchbiegungsmessungen sind im Artikel dargestellt.

1. INTRODUCTION

Administration building within an industrial complex in Arandjelovac, Yugoslavia, was constructed in 1978. During construction, and very soon after the building was furnished, appearance of cracks and appreciable deformations of the structure was noted. The condition of the building got worst in course of time, caused fear and uncertainty, until the owner has decided to call for help.

2. LAYOUT OF THE BUILDING

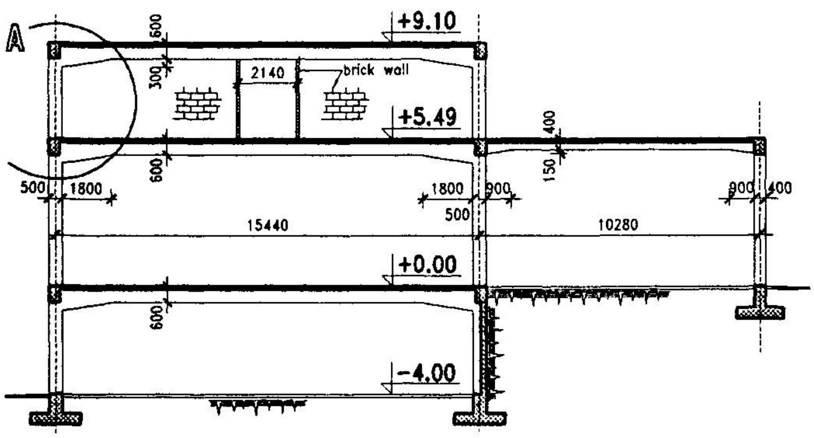

Fig. 1 Disposition of a transverse structure frame

The building is a three-story reinforced concrete frame structure with an annex, with dimensions 73,440 x 26,160 mm in plan, and clear story heights 3,400 + 4,890 + 3,061 mm. The span of the transverse frames, mutually spaced at 5,200 mm, is 15,440 mm in the main bay and 10,280 mm in the annex, Fig. 1.

Frame beams are haunched, of rectangular cross-section, dimensions 400 x 600 mm in the main bay and 400 x 400 mm in the annex. Frame columns, dimensions 400 x 500 mm are founded on strip foundations. Floor structure consists of semi-prefabricated solid reinforced concrete one-way slab, total thickness 120 mm.

Basement of the building is used for workshops, a restaurant is on the ground floor level, while the first floor is occupied by offices.

3. DESCRIPTION OF DAMAGES

The most outstanding cracks, up to 0.5 mm wide, appeared in the structure beams. In the span cracks spread in vertical direction along the entire depth of the beam, up to the roof slab. Towards the ends of the beams, the cracks gradually incline up to an angle of approximately 60 degrees regarding the horizontal plane, Fig. 2.

Appearance of inclined cracks at the top of the short columns in the top story, up to 12 mm wide, indicates failure of the column, Fig. 2. Due to increased deflections of frame beams, up to 60 mm in case of roof beam (span-deflection ratio 255), the functionality of the structure was violated, specially at the top story: appearance of doors and windows distortion and jamming, discomfort while walking, separation of partition brick walls from frame beams and columns, in some places more than 1 cm which violated their lateral stability as well as functionality of the installations. Excessive deflections of the roof beams caused loss of the draining layer slope and therefore retaining of water on the roof.

4. INVESTIGATION OF DAMAGE CAUSES

Adopted small cross-section dimensions of transverse frame elements, especially of beams (span-depth ratio 25.7), even visually give an impression of very slender, deformable frame. Emphasized deflections of slender beams could be accepted at the roof level, with a choice of initially larger slope of the roof. However, at the level of restaurant and offices, provision of functionality of the structure required a more careful choice of girder dimensions and check of deformations.

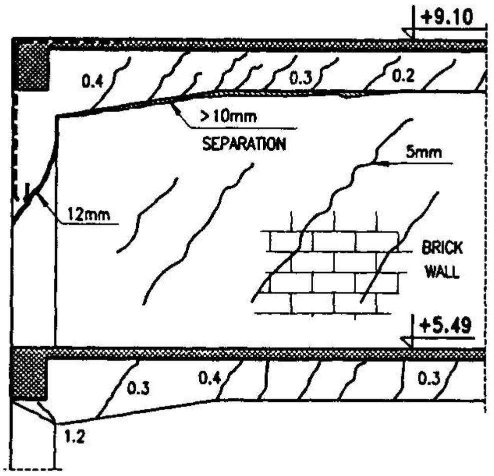

Fig. 2 Characteristic detail 'A' of damages

Subsequent structural analysis showed that, due to omission in load estimation and pattern, safety coefficients regarding the ultimate limit state in the large part of the structure are considerably smaller than required, and that the cross-sections are under-reinforced. For example, original structural analysis was carried out with assumption of uniformly distributed load, which underestimated "concentrated" effect of load from partition brick walls 120 mm thick, which are perpendicular to transverse frames in the corridor zone, Fig. 1. During service of the building, permissible tensile stresses in reinforcement in a large number of beam and column sections were exceeded, and notable cracks and deformations appeared.

The intensity of columns loading was additionally underestimated, since the stiffness of the beam haunches was not taken into account. As it is common in design of buildings, control of shear strength of columns was not carried out. In case of the third floor 'shallow frame' columns, that was an unacceptable oversight. Inspection of performed column reinforcement details showed that the tensioned reinforcement at the column-roof beam joint was not anchored deep enough into the column, which created conditions of "pulling out" the beam from the column, Fig. 2. At the same time, the lateral reinforcement of the columns is not sufficient to resist developed large shear forces, therefore, failure of column, decrease of rigidity of this story and increase of deformations of roof frame beam occurred.

Testing concrete quality by core drilling showed that, in some regions of the structure, concrete of lower quality than required by original design had been obtained.

Conclusion was that the damages of the structure as well as functional problems occurred due to uncarefull selection of cross-section dimensions, inadequate structural analysis and reinforcement detailing, as well as pure quality of workmanship.

5. THE STRUCTURE STRENGTHENING

While selecting the method of structure strengthening, three basic concepts were considered:

- Structure strengthening in existing stress and deformation condition. In that case strengthening elements are activated for additional loads only;

- simultaneously structure strengthening with partial correction of existing stress and deformation condition - strengthening elements are activated for a part of existing loads as well, by temporary raising of primary structure by hydraulic jacks, for example;

- correction of existing condition of stresses and deformations, applying additional permanent external forces, by tensioning the existing structure by prestressing tendons - "active strengthening".

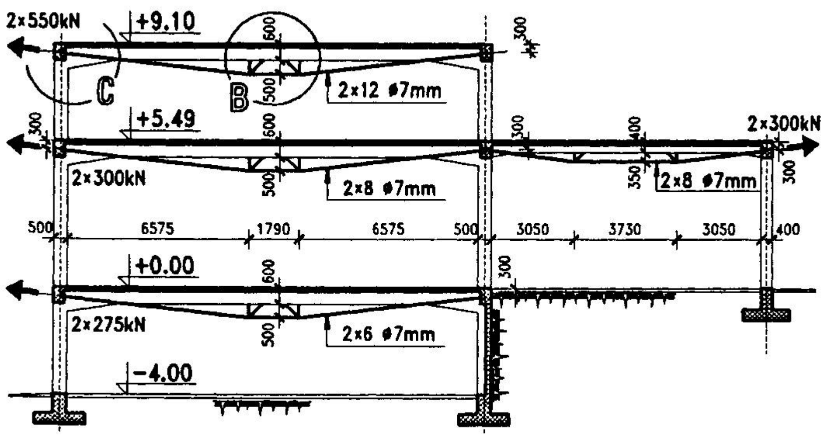

Fig. 3 Tendon layout

Two latter solutions were presented to the owner as technically acceptable. Taking into account the Owner's particular requirement of the least possible disturbance of the building service during strengthening works, the third concept was accepted as the most rational solution.

Correction of stresses and deformations of the structure as a whole was performed by shoring each frame beam with two prestressing tendons (6-12)#7, Yugoslav IMS system, conducted below the beams, Fig. 3. Required sag of 500 mm between the tendons and the beam soffit in the main span, and 350 mm in the annex was provided by inserting steel deviators - "chairs", constructed of steel tubes 63.5 mm in diameter, Fig. 4. Deviator verticals are placed in such a way that tendons equivalent forces

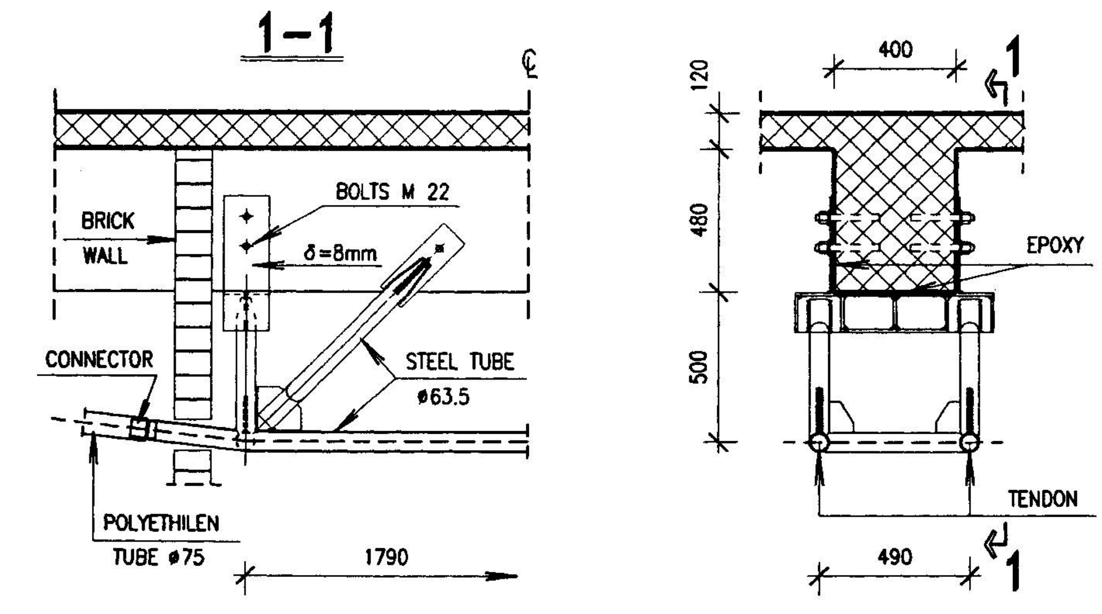

Fig. 4 Deviator detail 'B'

counteract the weight of two heavy partition walls in the top floor corridor. Tendon deviation over the central column at the connection with the annex was achieved by performing the cables through circular segment of steel tube. Tendon anchoring was carried out by a steel anchor box at the external side of the columns, Fig. 5. At straight segments the tendons are performed through polyethilen tubes, which are connected to steel elements after tensioning the tendons. Tubes and anchor boxes are injected with cement emulsion.

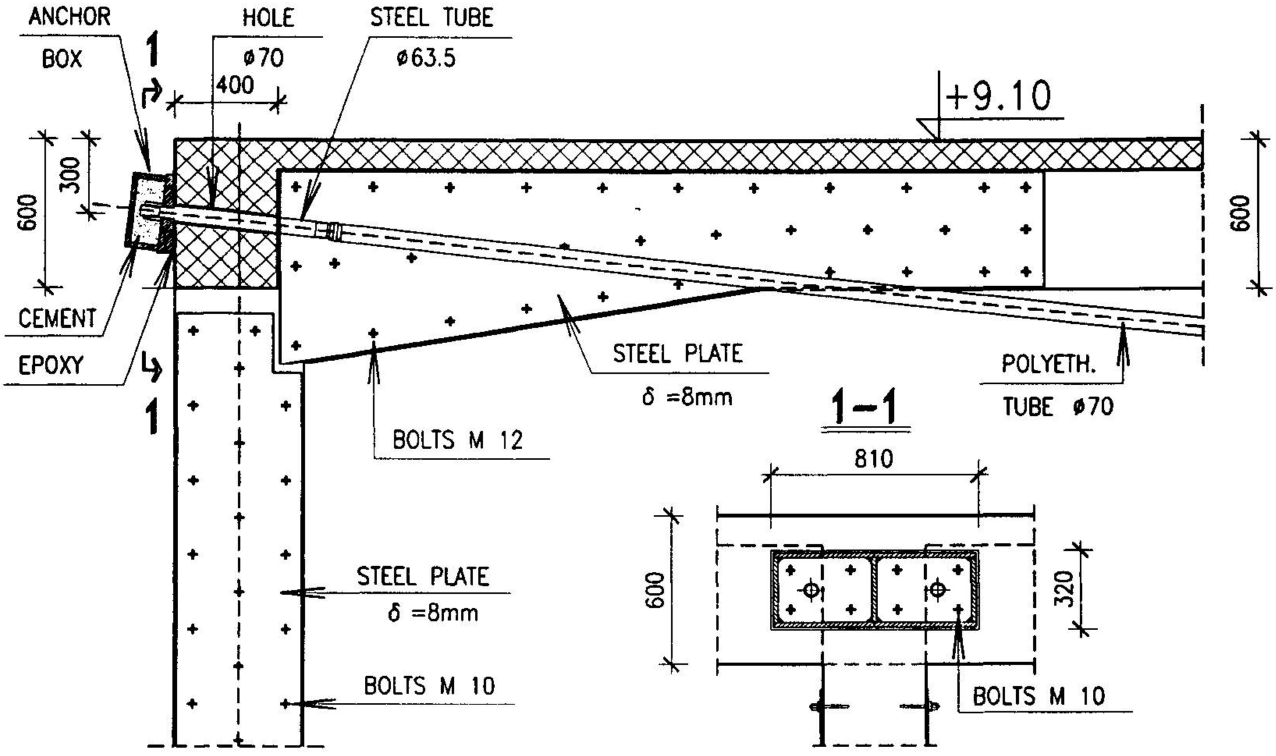

Fig. 5 Detail 'C' of tendon anchoring and lateral strengthening with steel sheets

Before tensioning of the tendons, injecting of cracks and local strengthening of beam and column ultimate shear capacity was carried out by well experienced method of glueing steel sheets onto lateral sides, by glue on the basis of low viscosity epoxy raisins with addition of filler [1]. Firm contact and glue spreading was achieved by bolts, Fig. 5.

The level of tensioning was so selected to fulfill the strengthening criterion that in all frame cross-sections required safety coefficient regarding ultimate limit state is provided. Order of tendon tensioning was from top to bottom story. Tendon tensioning was carried out by Institute for Testing of Materials of the Republic of Serbia - the IMS Institute.

6. RESULTS OF STRENGTHENING

During tendons' tensioning stage, IMS Institute carried out measurements and observations intended to verify accuracy of tendon forces as well as the structure response to actually realized forces, comprising, among other:

- measurement of change of the vertical displacement of the beams with mechanical deflection gauge during few days, up to stabilization (control of global tensioning effects and inertia of system response)

- measurement of strain change in steel elements - "chairs" and steel sheets (control of the friction effects and control of bond efficiency between steel sheets and concrete).

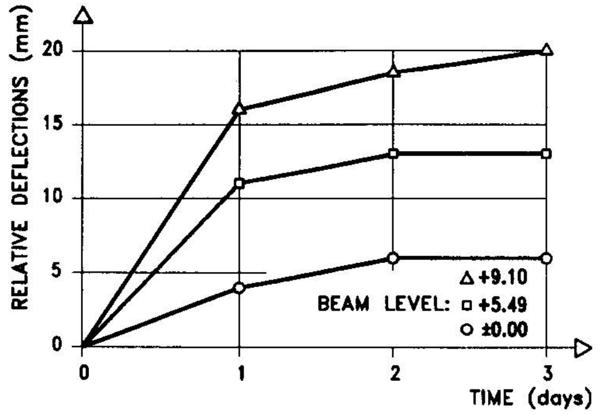

Fig. 6 - Stabilization of deflection in course of time

Results of performed measurements show good correspondence between measured and calculated relative deflections, with average stabilization time of three days following tensioning, Fig. 6. Measured values of change in strains of steel elements indicate well spreading of prestressing force, with expected level of friction, as well as that glued lateral steel sheets become activated.

On the basis of all measurements and observations, a conclusion was drawn that tensioning was successfully performed and that the structure response is within expected limits.

7. APPENDIX

Adopted tendon layout with steel chairs basically change the structure stiffness for additional loads, transforming the frame beams into the "two-chord system". But, utilization of the tendons with relatively small steel area tensioned to high stress level, as in presented case, the structure stiffness is only slightly increased, the basic static system practically does not alter. Control of ultimate bearing capacity of the strengthened structure shows that the effect of the tendons at the most appears as a particular case of external load. Rupture safety coefficient of the tendons actually depends only on the initial stress level during tensioning, since the stress variations in the tendons due to structure deformations under additional loads are negligible.

Concept of "active strengthening" by prestressing tendons was also used up to now by the authors of this paper in cases of bridge [2] and silo strengthening, when insufficient bearing capacity of the structure is prevailing. However, when simultaneous increase of the structural stiffness is also required, the two-chord concept can be also applied providing the greater stiffness of the lower chord by combining the structural - "passive" steel and prestressing tendons - "active" steel of the cross-section.

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